STEEL SPANDREL - BRACED TWO HINGED ARCH RAILROAD BRIDGE

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ARMOUR INSTITUTE OF TECHNOLOGY

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Design of a 120 foot steel
spandrel-braced two hinged

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D E S I G N O F A

120 FOOT STEEL SPANDREL-BRACED
TWO HINGED ARCH
RAILROAD BRIDGE

A THESIS

Presented by

E. L. Curren
A. L. Hess

To the

PRESIDENT AND FACULTY

of

ARMOUR INSTITUTE OF TECHNOLOGY

For the Degree of

BACHELOR OF SCIENCE IN CIVIL ENGINEERING

Having Completed the Prescribed Course of Study in

C I V I L E N G I N E E R I N G

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INTRODUCTION

* * * * * *

Arches are classified by the number of hinges which they have. There are three kinds of steel arches under this heading, namely: three-hinged, two-hinged and arches with no hinges. The hinges are situated in the lower chord at the abutments and crown. In the two-hinged arch the one at the crown is omitted.

The form of three-hinged arch that is generally used in bridges is composed of a curved or bent lower chord and a horizontal upper chord united by vertical and diagonal bracing as shown in figure on page #5. This type is called a spandrel-braced arch.

Another form of three-hinged arch is that of an arch rib with a solid web. This is a curved plate girder of uniform radical depth throughout. The axis is either parabolic or circular and the flanges are composed of angles and plates. Several of these ribs are placed at short distances apart and are connected by lateral bracing. The floor system is supported on these by vertical posts which are braced with sway bracing. No diagonal bracing is used in this type.

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Common forms of the two hinged arch bridge are the spandrel-braced and the arch rib, which are similar to the three hinged types of arches. The arch ribs may have either a solid or open web in this case.

The larger part of the deflection of a three-hinged arch is caused by temperature changes. This can be reduced by making the arch continuous at the crown, because the two halves of the arch are no longer free to turn at that point. The deformation of the arch, due to changes in temperature, is partially prevented in the two-hinged arch which produces stresses in the members. This form of arch is better adapted to railroad traffic than the three-hinged arch, because it forms a stiffer structure. This is especially true in the case of long spans.

Arches with no hinges have their ends bolted to the abutments so that no changes in direction can occur, which produces bending moments at the ends. This form of arch is stiffer and has higher temperature stresses than other kinds.

DESIGN OF A

STEEL TWO-HINGED SPANDREL-BRACED ARCH RAILROAD BRIDGE

Single Track Cooper E-50 Loading

Am. Ry. and M. of W. Assoc. Specifications

Curve of lower chord is parabolic.

The trusses are parallel to each other and are connected by sway bracing, and upper and lower lateral bracing.

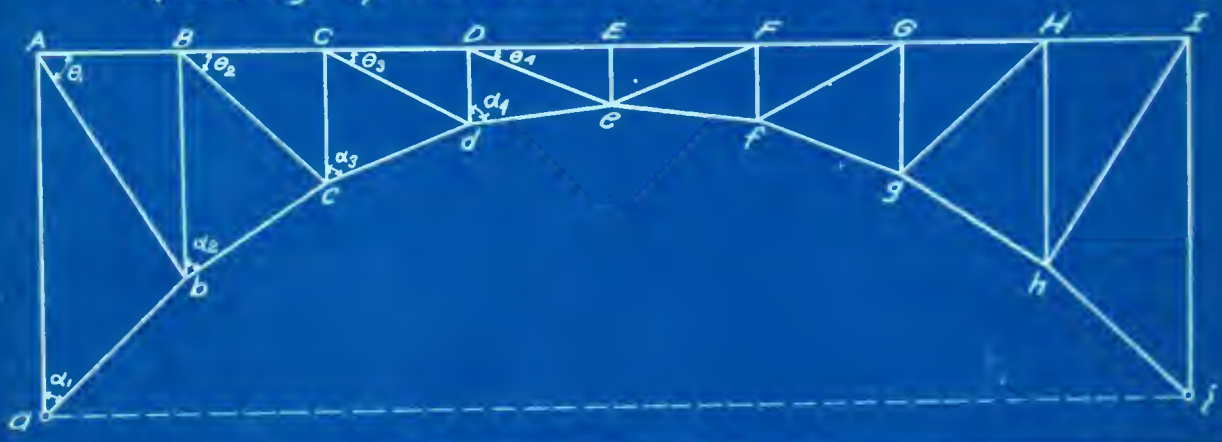
GENERAL DIMENSIONS

Span = 8 panels @ 15 ft. = 120 ft. c.-c. of Pins.

Rise of upper chord = 38 ft.

Rise of crown = 32 ft.

Spacing of trusses = 16 ft. c.-c.

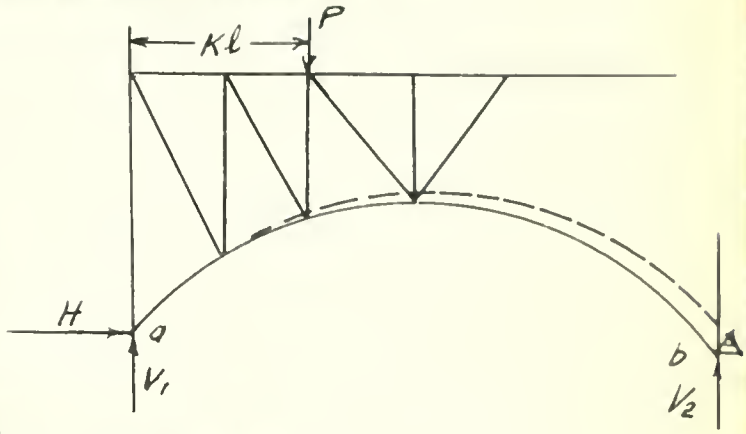


Angles		Rise of Lower Panel Points	Lengths of Members in ft.		
$\theta_1 = 58°00'$	$\alpha_1 = 47°00'$	b = 14'-0"	Aa = 38.0'	Ee = 6.0'	De = 16.155'
$\theta_2 = 43°00'$	$\alpha_2 = 56°20'$	c = 24'-0"	Bb = 24.0'	Ab = 28.302'	ab = 20.518'
$\theta_3 = 28°02'$	$\alpha_3 = 68°12'$	d = 30'-0"	Cc = 14.0'	Bc = 20.518'	bc = 18.028'
$\theta_4 = 21°47'$	$\alpha_4 = 82°25'$	e = 32'-0"	Dd = 8.0'	Cd = 17.0'	cd = 16.156'
					de = 15.133'

The equation of the parabola $y = ax^2$ was used to find the difference in length of vertical members, where $a = 2$ and x = number of panels from 'e'.

Derivation of Formula for H

Let L = length of any member.
A = area of any member.
S' = stress due to vertical reaction of unit load
T = " " " unit horizontal reaction



To find the value of H, it is assumed that the hinge (b) moves under the load P a distance Δ and is brought back again by the force H.

Therefore the displacement Δ due to P is equal to the displacement Δ due to H.

Total work in a member = $\frac{1}{2}$ S' (e + e' + e")

e = elongation due to unit horizontal force.
e' = " " " (H-1).
e" = " " " P

The work in any member due to unit horizontal force = $\frac{1}{2}$ S'e

$$e = \frac{TL}{AE}$$

$$K = \frac{1}{2} \frac{S'TL}{AE}$$

For all members

$$K = \frac{1}{2} \sum \frac{S'TL}{AE} = \frac{1}{2}(\Delta \times 1)$$

$$\Delta = \sum \frac{S'TL}{AE} \quad \text{(due to P)}$$

If T = stress in any member due to P

$V = TH$

Work in any member $= \frac{1}{2}\frac{V^2L}{EA}$ (Merriman Strength of Materials. Page 308)

$$\frac{1}{2}\Delta H = \frac{1}{2}\sum \frac{V^2L}{EA}$$

$$\Delta = \frac{1}{H}\sum \frac{V^2L}{EA} = H\sum \frac{T^2L}{AE} \quad \text{(due to H)}$$

Equating Δ (due to P) to Δ (due to H)

$$\sum \frac{S'TL}{AE} = H\sum \frac{T^2L}{AE}$$

$$H = \frac{\sum \frac{S'TL}{AE}}{\sum \frac{T^2L}{AE}}$$

For the preliminary values of H we assume constant values of A for all members which gives us the equation $H = \frac{\sum S'TL}{\sum T^2L}$

L O A D I N G

The weight of steel in a 125 ft. simple truss equals 3.888 kips 1 ft. of bridge and the weight of track for specified loading equals .575 kips 1 ft. of bridge. The weight of steel in an arch bridge is considered to be less than that of a simple truss of the same span. The weight of steel in the 120 ft. arch bridge was assumed at 3.425 kips 1 ft. of bridge. Therefore the dead load equals (3.425 + .575) or 4.00 kips 1 ft. of bridge, which is 30. kips per panel per truss.

The live load per foot of truss equals the weight of one Cooper E-50 engine divided by its length. Live load equals $\frac{177.5}{56}$ = 3.171 kips 1 ft. of truss = 47.6 kips 1 panel 1 truss.

METHOD OF PROCEDURE

The stresses S' in the members were calculated first, for a load of unity at the various panel points. S' is the stress in a member when the arch is considered as a simple truss, having only vertical reactions. The stress S' was only found for the load unity at the panel points on the left including the middle one, because the stress in a member on the right with the load at a panel point on right is the same as that for a corresponding member on the left and load at a similar panel point on left. The values of S' for the chords was found by passing a section through the panel and taking moments about the intersection of the diagonal and the other chord, and for the web members was found by resolving about the joints. Then the stresses T due to a horizontal force unity acting outward at the hinges was calculated for all members of the truss. The preliminary values of S' and T are given on Page #10. The values of S'TL where L is the length of the member in feet, and S' and T the corresponding stresses, were summed up for the different positions of the load unity giving the values Σ S'TL shown on page #12. Then the value of Σ T^2L was calculated. In the formula for H, given on Page #4, the value of E is constant

and drops out; L is considered in feet instead of inches because the 12 is also a constant and the values of A are taken as being equal. This gives an approximate formula for H., which is $\Sigma S'TL / \Sigma T^2L$. Substituting in this formula gives the preliminary values of H., which are the same for corresponding panel points on each side. Then the stress S in any member due to the load unity at the different panel points can be found from the equation $S = S' - HT$. The sum of the tensile and compressive values of S multiplied by the live panel load gives the live load tensile and compressive stresses for each member. The sum of all the values of S for each member multiplied by the dead panel load gives the dead load stresses. Then the dead load stress was added, algebraically, to the live load tensile and compressive stresses including impact, giving a maximum tensile or compressive stress or both for each member. From these stresses and the specified unit stresses, the preliminary areas can be found. An excess of area should be given each member to allow for wind and temperature stresses. These values of A were substituted in the formula $H = \Sigma \frac{S'TL}{A} / \Sigma \frac{T^2L}{A}$ giving a more exact series of the value of H. The final values of H were substituted for H in the formula $S = S' - HT$ in the same manner as before.

From these values and the temperature stresses the final maximum tensile and compressive stresses are obtained for each member. With these stresses and the wind stresses the final areas can be found.

STRESSES DUE TO LOADS AT EACH PANEL POINT.

Load at	A	B	C	D	E	Horizontal Load at (a)
ab		0	0	0	0	+1.368
bc		+.658	+.564	+.470	+.376	+1.902
cd		+.870	+1.735	+1.443	+1.155	+2.930
de		+1.180	+2.360	+3.550	+2.840	+4.79
ef		+.710	+1.420	+2.130	+2.840	+4.79
fg		+.290	+.578	+.869	+1.155	+2.93
gh		+.094	+.188	+.826	+.376	+1.902
hi		0	0	0	0	+1.368
AB		-.547	-.469	-.390	-.312	-.584
BC		-.805	-1.604	-1.340	-1.070	-1.715
CD		-1.171	-2.342	-3.520	-2.820	-3.750
DE		-1.250	-2.50	-3.750	-5.000	-5.330
EF		-1.250	-2.50	-3.750	-5.00	-5.330
FG		-.704	-1.404	-2.110	-2.82	-3.750
GH		-.268	-.536	-.804	-1.07	-1.715
HI		-.078	-.156	-.234	-.312	-.584
Ab		+1.03	+.866	+.737	+.590	+1.100
Bc		+.353	+.155	+1.298	+1.033	+1.548
Cd		+.415	+.838	+2.474	+1.980	+2.310
De		+.085	+.170	+2.480	+2.350	+1.700
eF		+.589	+1.181	+1.770	+2.350	1.700
fG		+.495	+.986	+1.480	+1.980	+2.310
gH		+.260	+.520	+.780	+1.033	+1.548
hI		+.147	+.295	+.442	+.590	+1.100
Aa	-1.00	-.875	-.750	-.625	-.500	-.935
Bb		-1.240	-1.057	-.886	-.707	-1.053
Cc		-.194	-1.3925	-1.160	-.933	-1.081
Dd		-.0315	-.063	-1.092	-.876	-.632
Ee		0	0	0	-1.00	0
Ff		-.218	-.438	-.655	-.871	-.632
Gg		-.232	-.462	-.696	-.933	-1.081
Hh		-.177	-.354	-.531	-.707	-1.053
Ii		-.125	-.25	-.375	-.50	-.935

$\Sigma ST L$

Pot	A	B	C	D	E	$\Sigma T^2 L$
ab		0	0	0	0	38.4
bc		22.56	19.0	16.15	12.92	65.3
cd		38.7	81.9	68.4	51.7	138.7
de		85.7	172.5	257.8	205.9	347.6
ef		51.5	109.8	154.5	205.9	347.6
fg		13.7	30.6	41.13	54.7	138.9
gh		3.27	1.35	9.7	12.92	65.3
hi		0	0	0	0	38.4
AB		4.79	4.11	3.42	2.74	5.12
BC		20.7	41.33	34.5	27.56	44.2
CD		38.6	132.0	198.	158.6	211.0
DE		100.0	200.0	300.	400.0	426.0
EF		100.0	200.0	300.	400.0	426.0
FG		39.6	79.1	118.8	158.6	211.0
GH		6.9	13.7	20.8	27.56	44.2
HI		.69	1.37	2.06	2.74	5.12
Ab		31.6	27.6	22.96	18.38	34.26
Bc		11.2	49.3	41.2	32.8	49.15
Cd		14.4	32.94	97.4	77.9	90.9
De		2.16	4.68	6.81	64.6	46.7
eF		15.0	32.46	48.6	64.6	46.7
fG		17.18	38.8	58.2	77.9	90.9
gH		8.25	16.5	24.73	32.8	49.15
hI		4.6	9.19	13.76	18.38	34.26
Aa	35.5	29.6	26.62	22.2	17.73	32.4
Bb		31.4	26.72	22.4	17.9	26.66
Cc		29.4	21.1	17.59	14.43	16.4
Dd		.16	.24	5.51	4.4	3.19
Ff		1.1	1.66	3.3	4.4	3.19
Gg		3.51	7.0	10.55	14.43	16.4
Hh		4.48	8.87	13.43	17.9	26.66
Ii		4.44	8.89	13.3	17.73	32.4
Total.	35.5	799.83	1404.37	1947.5	2214.12	3152.06

Preliminary values of H

Load at A $\quad H = \dfrac{35.5}{3152.36} = .01124$

Load at B $\quad H = \dfrac{789.83}{3155.96} = .253$

Load at C $\quad H = \dfrac{1404.37}{3155.96} = .446$

Load at D $\quad H = \dfrac{1947.5}{3155.96} = .618$

Load at E $\quad H = \dfrac{2214.12}{3157.96} = .707$

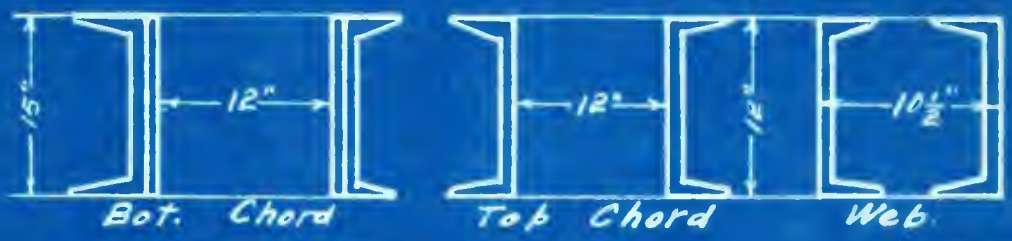
PRILIMINARY STRESSES IN MEMBERS.

S = S' − HT.

Q	ab	bc	cd	de	AB	BC	CD	DE	Ab	Bb	Cd	De	Aa	Bb	Cc	Dd	Ee
A	-.008	-.011	-.016	-.027	+.003	+.009	+.021	+.030	-.006	-.009	-.013	-.010	-.495	+.006	+.006	+.004	0
B	-.346	+.177	+.129	-.030	-.399	-.371	-.221	+.098	+.752	-.039	-.170	-.345	-.639	-.974	+.080	+.128	0
C	-.610	-.284	+.420	+.224	-.209	-.839	-.672	-.122	+.395	+.860	-.192	-.588	-.333	-.587	-.910	+.219	0
D	-.845	-.704	-.368	+.590	-.030	-.231	-1.204	-.36	+.056	+.342	+1.046	-.802	-.048	-.236	-.492	-.702	0
E	-.968	-.966	-.911	-.054	+.100	+.142	-.174	-1.24	-.188	-.059	+.350	+1.150	+.161	+.038	-.169	-.426	-1.00
F	-.845	-.895	-.942	-.830	+.126	+.255	+.206	-.360	-.239	-.178	+.052	+.720	+.203	+.119	-.028	-.265	0
G	-.610	-.471	-.728	-.716	+.104	+.229	+.266	-.122	-.196	-.170	-.044	+.423	+.167	+.116	+.020	-.156	0
H	-.346	-.377	-.451	-.502	+.069	+.166	+.246	+.098	-.131	-.132	-.090	+.159	+.121	+.090	+.043	+.058	0
I	-.008	-.011	-.017	-.027	+.003	+.009	+.021	+.030	-.006	-.009	-.013	-.010	+.005	+.006	+.006	+.004	0
+	.000	+.177	.558	.814	.405	.810	.760	.256	1.203	1.202	1.448	2.452	.657	.375	.155	.413	0
−	4.586	3.719	3.433	2.186	.638	1.491	2.271	2.204	.766	.596	.522	1.755	1.515	1.797	1.599	1.548	1.00
L +	0	8.43	26.54	38.76	19.27	38.55	36.2	12.18	57.3	57.25	68.9	116.7	31.23	17.82	7.38	19.65	0
L −	2.18	176.8	163.1	104.1	30.36	71.0	108.0	105.0	36.4	28.37	24.82	83.5	72.1	85.5	76.0	73.6	47.6
I +	0	8.02	24.1	35.2	15.4	30.82	30.2	10.59	49.85	52.1	59.9	97.1	25.5	14.25	6.16	16.38	0
I −	155.6	131.0	125.5	80.1	26.4	61.8	90.0	84.2	29.13	21.8	19.88	69.60	61.4	74.4	63.4	61.4	45.3
D	-137.58	-106.26	-86.25	-41.16	-6.99	-20.43	-45.33	-58.44	+13.11	+18.18	+27.78	+20.91	-25.74	-42.66	-43.32	-34.05	-30.
N +				32.8	27.68	48.94	21.07		120.26	127.53	156.58	234.71	30.99				
M −	311.18	404.16	374.85	225.36	63.85	153.23	243.33	247.64	52.42	31.99	16.92	132.19	159.24	202.56	193.72	169.05	122.9

PRELIMINARY AREAS.

Gusset Plates $\frac{3}{4}$" Thick.



Mem.	Gross Area - □"	Section	Required Area □"
ab	43.61	2-15"X55# Cs 2 Plates $\frac{3}{8}$"X16"	39.0
bc	34.775	2-15"X40# Cs 2 Plates 15"X$\frac{3}{8}$"	30.4
cd	32.36	2-15"X55# Cs	28.04
de	23.52	2-15"X40# Cs	15.9
AB	20.58	2-12"X35# Cs	13.9
BC	20.58	2-12"X35# Cs	13.9
CD	26.64	2-12"X30# Cs 2-12"X$\frac{3}{8}$" Plates	17.60
DE	26.64	2-12"X30# Cs 2-12"X$\frac{3}{8}$" Plates	17.6
Aa	19.80	2-15"X33# Cs	16.4
Bb	23.52	2-12"X40# Cs	19.0
Cc	17.64	2-10"X30# Cs	15.4
Dd	14.70	2-9"X25# Cs	12.2
Ee	11.76	2-9"X20# Cs	8.52
Ab	14.7	2-9"X25# Cs	9.15
Bc	14.7	2-9"X25# Cs	8.97
Cd	14.7	2-10"X25# Cs	10.31
De	23.52	2-12"X40# Cs	19.7

$$\Sigma = \frac{s'rL}{A}$$

P at	A	B	C	D	E	$\Sigma \frac{r^2L}{A}$
ab	0	0	0	0	0	.78
bc		.645	.543	.461	.38	1.87
cd		1.197	2.532	2.117	1.69	4.29
de		3.65	7.34	10.955	8.75	14.78
ef		2.192	4.66	6.57	8.75	14.78
fg		.423	.946	1.272	1.69	4.29
gh		.093	.210	.278	.380	1.87
hi		0	0	0	0	1.78
AB		.233	2.0	.166	1.33	.249
BC		1.007	2.01	9.68	13.40	2.15
CD		1.449	4.75	7.43	5.93	7.91
DE		3.76	7.5	11.27	15.0	16.0
EF		3.75	7.5	11.27	15.0	16.0
FG		1.487	2.97	4.45	5.93	7.91
GH		.259	0.666	1.011	1.34	2.15
HI		.034	.067	.10	1.33	.249
Ab		2.15	1.88	1.56	1.25	2.334
Bc		.761	3.355	2.803	2.232	3.35
Cd		.98	2.292	6.64	5.30	6.19
De		.092	.199	.29	2.75	1.088
eF		.638	1.38	2.07	2.75	1.988
fG		1.169	2.64	3.96	5.30	6.19
gH		.561	1.123	1.68	1.232	3.35
hI		.313	.626	.936	1.25	2.334
Aa	.815	1.495	1.345	1.120	.795	1.586
Bb		1.337	1.562	.952	.611	1.135
Cc		.167	1.198	.996	.819	.93
Dd		.011	.016	.376	.300	.217
Ff		.075	.113	.225	.300	.217
Gg		.199	.397	.598	.819	.930
Hh		.191	.378	.571	.661	1.135
Ii		.224	.449	.671	.795	1.586
Total	.815	30.547	60.997	84.498	94.66	131.518

Final Values of K

Load at A $\quad K = \dfrac{.815}{131.518} = .00615$

Load at B $\quad K = \dfrac{30.547}{131.518} = .23$

Load at C $\quad K = \dfrac{60.097}{131.518} = .484$

Load at D $\quad K = \dfrac{84.408}{131.518} = .643$

Load at E $\quad K = \dfrac{94.66}{131.518} = .72$

Temperature Stresses

The change in length of the span due to a temperature change of 75° divided by the horizontal displacement Δ of the right hinge due to a horizontal force of unity acting outward equals the horizontal reaction "H" caused by the temperature change. Then the temperature stress in each member "S" equals HT, where "T" is the stress in each member due to the unit horizontal force.

The change in length of span due to a temperature change of 75° = etl when

e, is the coefficient of expansion = 0.000,0065

t, is the rise in temperature = 75°

l, is the length of the span in inches = 120 x 12 = 1440

$$\therefore \text{etl} = .000,0065 \times 75 \times 120 \times 12 = .702 \text{ inches}$$

$$\Delta = H\Sigma \frac{T^2L}{AE} = \frac{131,518 \times 12}{2,9000,000} = .000\,0544$$

$$\therefore H = \frac{.702}{.600,0544} = 12.904 \text{ kips}$$

Temperature stress "S" = HT

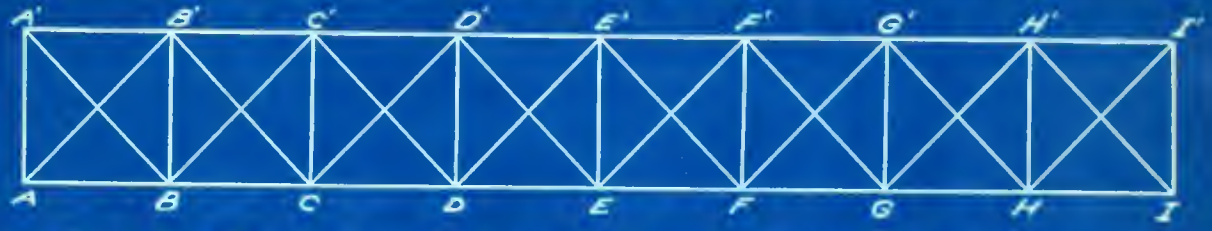
Values of "S" in kips

ab	=	17.65	AE	=	7.48
bc	=	24.50	BC	=	22.10
cd	=	37.80	CD	=	48.30
de	=	62.70	DE	=	68.80
Ab	=	14.2	Ad	=	12.05
Bc	=	19.95	Bb	=	13.01
Cd	=	29.80	Cc	=	13.92
De	=	21.95	Dd	=	8.15
			Ee	=	0.0

DESIGN OF UPPER LATERAL SYSTEM

Lateral force on loaded chord = 200 + 500 = 700#/lin. ft.

Panel load = $\frac{700 \times 15}{1000}$ = 10.50 Kips, considered as moving.



Span = 8 panels @ 15' = 120'-0" Spacing of trusses = 16'-0"

STRESSES in Kips

A'B = 50.30 C'D = 26.961 AB = 34.453 CD = 59.062

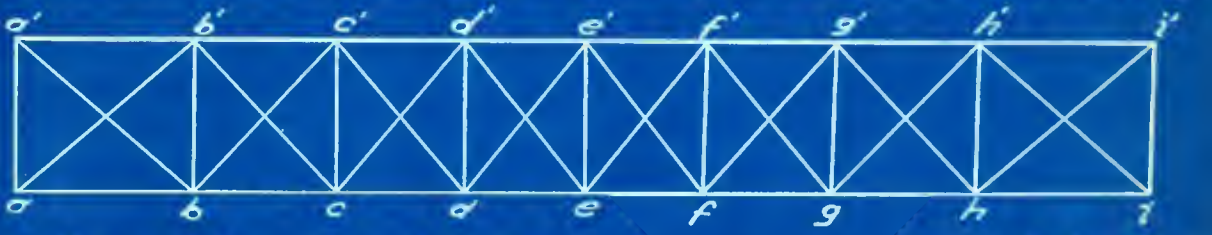
B'C = 37.745 D'E = 17.974 BC = 34.453 DE = 73.830

The diagonals are designed as tension members and are composed of 2 - $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$ angles

DESIGN OF LOWER LATERAL SYSTEM

Lateral force on unloaded chord = 200#/lin. ft.

Panel load = $\frac{200 \times 15}{1000}$ = 3.0 Kips, considered as moving



Considered as being spread out in horizontal plane

STRESSES in Kips,

ee' = 2.4	a'b = 17.062	ab = 13.46
dd' = 3.9	b'c = 11.913	bc = 13.46
cc' = 5.9	c'd = 8.384	cd = 21.91
bb' = 7.9	d'e = 5.370	de = 26.46
aa' = 10.5		

For diagonals and verticals use 2 - $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$ angles

FINAL STRESSES IN MEMBERS.

s = s'-HT.

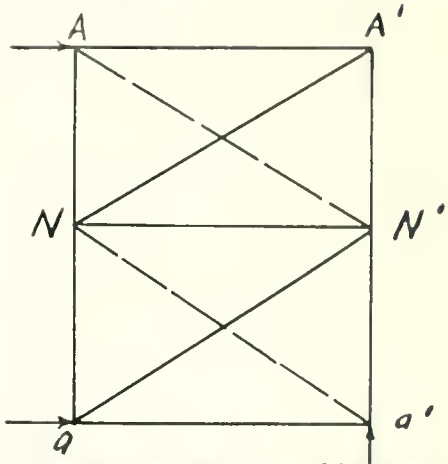
Pt	ab	bc	cd	de	AB	BC	CD	DE	Ab	Bc	Cd	De	Aa	Bb	Cc	Dd	Ee
A	-.004	-.006	-.009	-.015	+.002	+.005	+.012	+.016	-.003	-.005	-.007	-.005	-.497	+.003	+.003	+.002	0
B	-.318	+.216	+.190	+.079	-.411	-.407	-.300	-.012	+.774	-.008	-.121	-.310	-.658	-.995	+.057	+.116	0
C	-.635	-.319	+.375	+.139	-.198	-.808	-.602	-.030	+.375	+.832	-.232	-.619	-.316	-.568	-.891	+.230	0
D	-.879	-.752	-.437	+.470	-.013	-.239	-1.110	-.328	+.030	+.303	+.989	-.845	-.024	-.209	-.464	-.691	0
E	-.985	-.993	-.953	-.605	+.108	+.163	-.120	-1.164	-.202	-.079	+.320	+1.128	+.173	+.052	-.154	-.416	-1.00
F	-.879	-.940	-1.101	-.950	+.143	+.297	+.300	-.328	-.265	-.215	-.005	+.677	+.226	+.147	0	-.254	0
G	-.635	-.694	-.782	-.801	+.115	+.260	+.336	-.030	-.216	-.198	-.084	+.392	+.184	+.135	+.040	-.145	0
H	-.318	-.347	-.370	-.401	+.057	+.130	+.167	-.012	-.109	-.101	-.041	+.194	+.092	+.068	+.020	-.071	0
I	-.004	-.006	-.009	-.015	+.002	+.005	+.012	+.016	-.003	-.005	-.007	-.005	+.003	+.003	+.003	+.002	0
+	0	.216	.565	.688	.427	.820	.827	.032	1.179	1.135	1.309	2.391	.678	.408	.123	.350	0
−	-4.657	4.057	3.661	2.787	.622	1.454	2.132	1.904	.798	.611	.497	1.874	1.495	1.772	-1.509	1.577	1.00
L +	0	10.28	26.9	32.73	20.3	39.0	39.4	1.52	56.0	54.0	62.2	113.8	32.22	19.4	5.85	16.64	0
L −	221.8	193.0	174.1	132.7	29.6	69.2	101.3	90.6	38.	29.08	23.64	89.15	71.1	84.4	71.7	75.0	47.6
I +	0	9.78	24.43	28.42	16.22	31.2	32.8	1.45	48.7	49.1	56.5	95.0	25.8	15.5	4.88	14.48	0
I −	159.6	142.9	134.0	105.7	25.76	60.2	84.5	71.1	30.4	22.37	18.2	74.4	61.9	73.4	62.4	60.	45.3
D	-139.8	-115.2	-92.88	-62.97	-5.85	-19.02	-39.15	-56.16	+11.43	+15.72	+24.36	+15.51	-24.51	-40.92	-41.58	-36.81	-30.
T	±17.65	±24.5	±37.8	±62.7	±14.2	±19.95	±29.8	±21.95	±7.48	±22.10	±48.30	±68.80	±12.05	±13.61	±13.92	±8.15	0
M +					30.67	51.18	33.05		116.13	118.82	143.06	224.31	33.51				
M −	521.2	451.1	400.98	301.37	61.21	148.42	224.95	212.86	56.97	35.73	17.48	148.04	157.51	198.72	175.68	171.81	122.9
W	±13.46	±13.46	±21.19	±26.46	±34.45	±34.45	±59.06	±73.83					±99.75				

FINAL AREAS.

	Section	Actual Area	Req. Areas
ab	2-16"x55# [s 2-Plates 15"x⅜	43.61	42.9
bc	2-45#x15" [s 2-⅜"x15" Plates	37.73	35.4
cd	2-15"x 55# [s	32.36	31.
de	2-50#x15" [s	29.42	28.2
AB	2-10"x20# [s	11.76	7.2
BC	2-10"x 30# [s	17.64	14.92
CD	2-12"x35# [s	20.58	20.3
DE	2-12"x35# [s	20.58	19.02
Aa	2-15"x 35# [s	20.58	20.22
Bb	2-12"x40#	23.52	21.2
Cc	2-10"x30#	17.64	16.3
Dd	2-9"x 25#	14.7	13.0
Ee	2-9"x 20#	11.76	9.5
Ab	2-9"x25#	12.18	9.29
Bc	2-9"x25#	12.18	9.24
Cd	2-10"x25#	12.46	11.95
De	2-12"x40#	23.52	22.46

Design of Sway Bracing

The stress due to wind is transmitted through the upper and lower latteral systems to the end. From there it is carried by the end sway bracing to the lower abutment. Four panel loads from the upper and lower latteral systems are considered as being applied at "A" and "a" respectively. The stresses in the members are found by taking moments about the joints. Because of the relative dimensions of the rectangle, it is divided into two panels as shown in figure.



A vertical reaction is produced at a' which is found by taking moments of the external forces about "a". This force causes a stress in the vertical A'-a'. The sway bracing at panel points "B" and "C" is composed of one panel each. Owing to the depth of the truss of D and F, no sway bracing is placed there. The stresses in the intermediate panels are nominal.

Aa = 38' AA' = 16' Load at A = 42 kips
Load at a = 12 kips P = 99.75 kips

Stresses in kips

AA'	= -42.0	N'a	= +65.1
A'N	= +65.1	aa'	= -54.0
NN'	= -42.0	a'N'	= -99.75

Design

NN' use 2 - 5" x 3" x $\frac{7}{16}$" angles

dd' " 2 - 5" x 5" x $\frac{1}{2}$" "

Diagonals, use 2 - 4" x 3" x $\frac{3}{8}$" angles

Intermediate Diagonals
use 2 - $3\frac{1}{2}$" x 3" x $\frac{3}{8}$" angles

Design of Stringers

Panel length = 15'-0"
Stringers are spaced 8'-0" C.-C.
Dead load due to ties and rails = 288 #1 [illegible] ft.
Dead load due to stringer = 140 #1 " "
Total dead load = 428 #1 " "

Maximum dead load moment = 13,[illegible] ft.lbs
" live " " = 170,[illegible] " "
Impact = 1[illegible] " "
Total bending moment = [illegible] " "

Maximum dead load end shear = 3[illegible] #
" live " " " = [illegible] #
Impact = 49780 #
Total end shear = 105[illegible] #

Required flange area = 1[illegible].61 sq.in. [illegible] flange area equals 1[illegible].96 sq.in.
The stringers are composed of four angles 6" x 6" x $\frac{3}{8}$" and one plate $23\frac{1}{4}$" x $\frac{1}{2}$".
The depth back to back of angles equals 24".
The rivet pitches from the end to middle of stringer are as follows: 3'0" of $2\frac{1}{4}$"; 2'-0" of 3" and 1'-[illegible]" of 4"; and 1'-$4\frac{1}{2}$" of 5" spaces

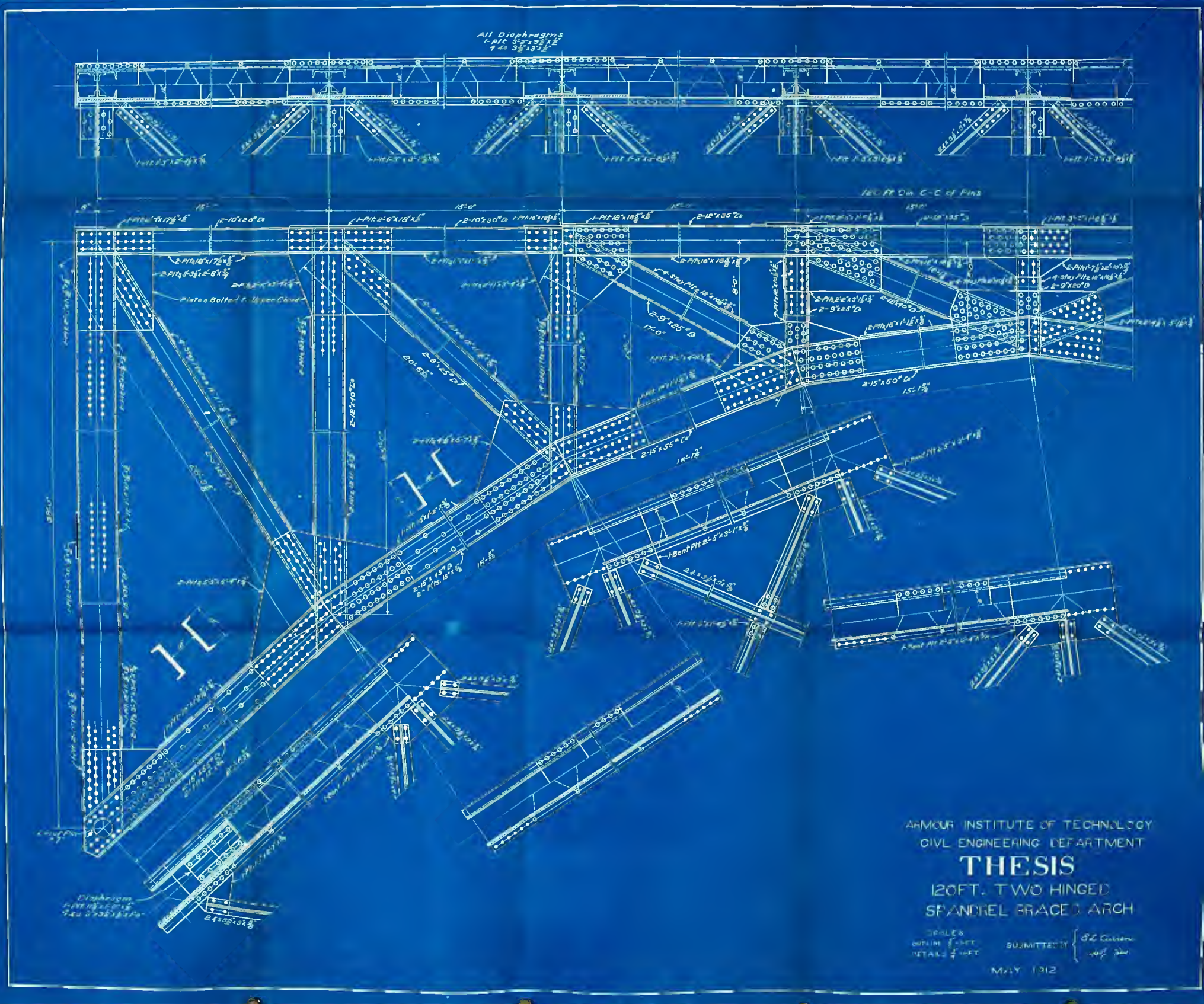
Design of Floor Beam

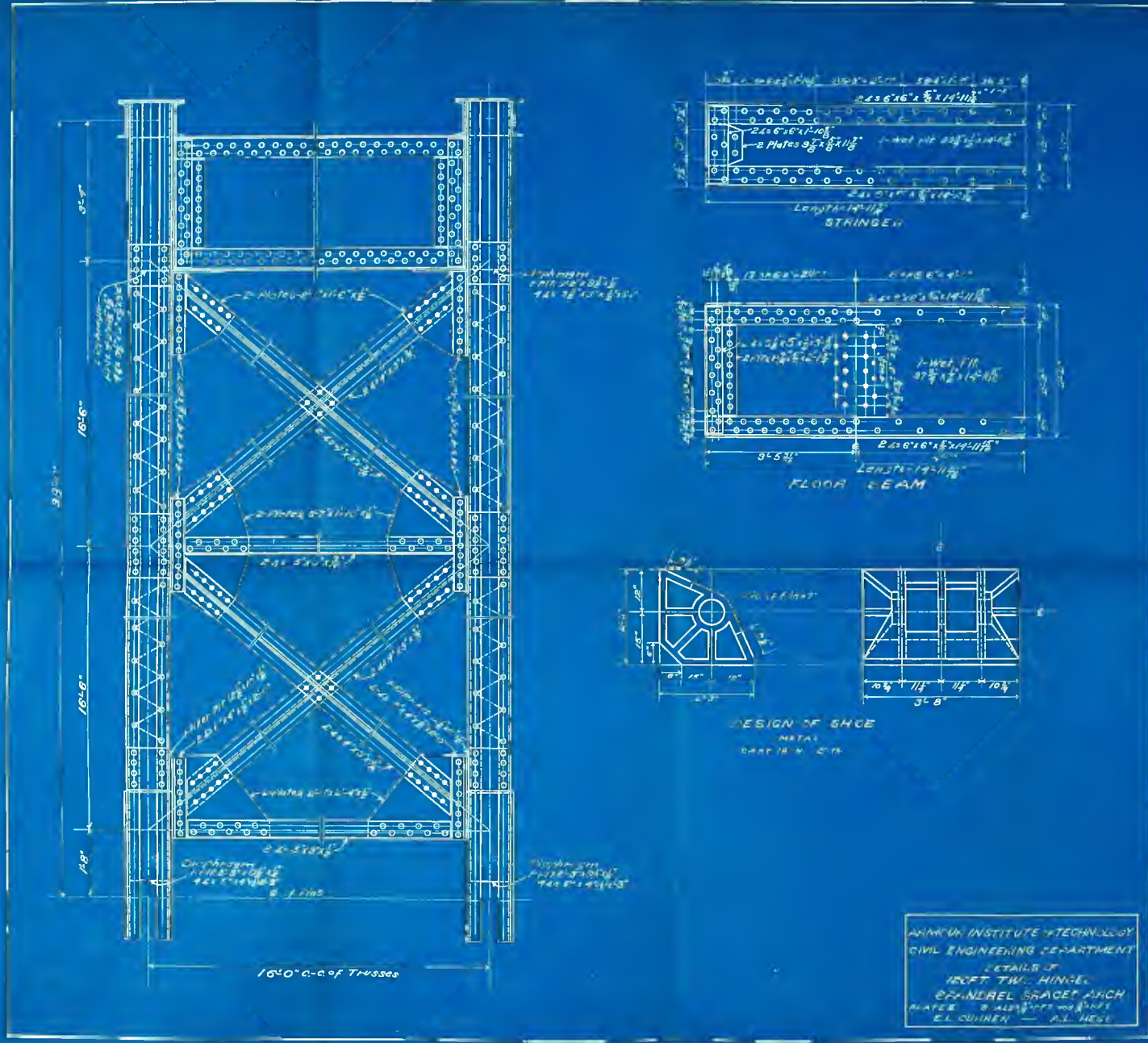
Length of floor beam equals 16'-0"
Dead load reaction = 6740.#
Live " " = 70058.#
Impact = 66510.#
Total reaction = 143308.#

Maximum bending moment = 573[illegible] ft. lbs.
The section is composed of four angles, 6" x 6" x $\frac{3}{8}$" and one plate $37\frac{1}{4}$" x $\frac{1}{2}$".
The distance back to back of angles equals 38".
The rivet spacing from end to center is as follows: 4'-0" of $2\frac{7}{16}$ inch and 4'-0".

Design of Pin at Hinge

Shear = 332.3 kips
Required area for shear = [illegible].7 sq. in.
Use a 7" pin
Required bearing = 1.075"
Use $\frac{1}{4}$" gusset plates.
Actual bearing = 200"
Use 2" pin plates on shoe.
Bending moment = 658. in kips
Required bearing area of shoe for vertical pressure = [illegible] sq. and for horizontal pressure = 659 sq. in.


All Diaphragms
120 Ft. Dia. C-C of Pins
Plates Bolted to Upper Chord
Diaphragm
ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
THESIS
120FT. TWO HINGED
SPANDREL BRACED ARCH
SCALES
SUBMITTED BY
MAY 1912


STRINGER
FLOOR BEAM
DESIGN OF SHOE
16'-0" c.-c. of Trusses
ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
DETAILS OF
100FT. TWO HINGED
SPANDREL BRACED ARCH
E.L. CURREN — A.L. HESS